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American Welding Society

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JOURNAL

of the

AMERICAN WELDING SOCIETY

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AMERICAN WELDING SOCIETY

ESTABLISHED 1919

JOURNAL

Volume 9 November, 1930 Number 11

Code for Fusion Welding and Gas Cutting in Building Construction

The A. W. S. announces the publication of 1930 Edition of Code 1, Part A, Code for Fusion Welding and Gas Cutting in Building Construction.

This edition, quoting from its Preface, "includes an Appendix whose use obviates the necessity of consulting the publications of the American Bureau of Welding cited in the 1928 Edition; and the body of the Code has been reworded to correspond."

"The matter given in the Appendix is primarily a digest of the publication previously referred to, with additional explanatory matter where needed to make it more applicable to Code purposes."

The 1930 Edition, as explained by the Committee on Building Codes under the A. W. S. is, in other respects, essentially as published in 1928, except for minor changes in the wording of three paragraphs in the body of the Code.

The changes apply to

Sec. 3, Par. 1, wherein the Committee has limited the application of structural welding to a cited specification for base metal parts.

Sec. 4, Par. 1, wherein a design criterion for filled welds, placed transverse to the direction of stress, has been included, and

Sec. 5, Par. 5, wherein beveling of butt joints which have only one edge beveled for welding have been definitely classified.

Copies of the 1930 Edition, Code 1, Part A, are available at A. W. S. headquarters, 33 West Thirty-ninth Street, New York City.

Section Meetings

The national Meetings and Papers Committee of the Society is compiling a list of prospective authors of papers for section meetings. Members who desire to present papers before some local section should promptly notify headquarters of the availability of any data that they may have on anv phase of welding.

New Orleans Adopts Welding Code

The city of New Orleans on Sept. 2 incorporated in the Municipal Code of that city provision for welding steel building frames. The American Welding Society for Fusion Welding and Gas Cutting was adopted with any subsequent amendments and additions thereto.

Welding Program Feature of International Acetylene Association

The Thirty-First Annual Convention of the International Acetylene Association was held in the Congress Hotel, Chicago, November 12, 13, 14, 1930.

The major portion of the program was devoted to oxy-acetylene welding and cutting and special sessions were also arranged on metal working industry, building construction, building piping, and the distribution of oil, gas and water.

A cordial invitation to attend this Convention was extended to everyone interested in oxy-acetylene welding and cutting or in the various industries which use the oxy-acetylene process.

Among the papers presented at this Convention were the following:

Production Welding by the Oxy-Acetylene Process; Oxy-Acetylene Committee Report; Experiments on Gas Welded Structural Joints; Possibilities of Welding in Private Homes; Securing Acceptance of Structural Welding by Architects, Structural and City Engineers; Welding Piping Systefs in Montgomery Ward & Com-

pany Building in Chicago; Welding as Applied to Sprinkler Systems; Tube Turns; Welding and Cutting on Railroads; Truck Transportation; Oxy-Acetylene Welding in Local Gas Transmission and Distribution.

In Memoriam

At a meeting of the Executive Committee of the American Welding Society held in the Engineering Societies Building, New York, N. Y., on the fifth day of November, 1930, the following resolution was unanimously adopted:

The very unexpected and sudden death of Mr. James Harvey Edwards on August 14, 1930, removed from the American Welding Society one of its most prominent and useful members.

The American Welding Society, therefore, desires to record its sincere appreciation of his character as a man; of his unlimited kindly consideration for the viewpoint of others with whom he was associated; of his scientific attainments and accomplishments as an engineer; of his pioneering spirit and enthusiasm with which he undertook practical research work for the Society as Chairman of the Structural Steel Welding Committee; of the results accomplished through his efforts up to the time of his death and of their value to the study of fusion welding and the structural steel industry at large.

The American Welding Society further desires to record its regret that he could not have lived to see the completion of his work and the satisfaction of seeing the application of its results and the realization of dreams inspired by his engineering vision and enjoying the grateful appreciation of his fellow engineers for these valuable services, therefore, be it

Resolved, That these statements of appreciation be spread upon the records of the Society and copies be sent to the family as an expression of our sympathy at this time of mutual loss.

A. E. DOYLE, President.

Section Activities

Boston

The first meeting of the season of the Boston Section was held on Oct. 24 at the plant of the Austin-Hastings Co., Inc., Cambridge. Alexander Churchward, chief engineer of the Wilson Welder & Metals Co., gave a very interesting talk on the welding of non-ferrous metals, after which there were demonstrations of arc welding of the non-ferrous metals. About 160 attended this meeting. Buffet lunch was served after the lecture.

The November meeting will be held on the 14th at 7.30 p. m. in the plant of the Thomson-Gibb Electric Welder Co., Lynn, Mass. Papers on the following subjects will be read and discussed: Welding of Motor Magnet Frames; Seam Welding; Projection Welding as Applied in the Automobile Industry. The speaker will be M. Thomson, General Electric Co., and W. T. Ober, Thomson-Gibb Electric Welding Co. Demonstrations of the operation of manual and automatic butt, spot and seam welding machines will be shown.

Cleveland

The first of a series of interesting meetings arranged by the Cleveland Section was held on Thursday, Oct. 23, at Carnegie Hall. The speaker of the evening was Mr. Zorn of the Detroit Edison Co., and the title of his address was "Four Years of Welding in Power House Construction and Maintenance." This address was illustrated with several slides, showing various jobs which have been done under Mr. Zorn's supervision. The meeting was very well attended.

New York

"Structural Welding" will be the subject of the November meeting of the New York Section, which is to be held on the 18th at 7.45 p. m. in the Engineering Societies Building. "Present Status of Structural Welding Throughout the United States from the Standpoints of Application

and Regulation" will be presented by the Committee on Building Codes, A. W. S. "Progress of Structural Welding Regulation," by Prof. F. P. McKibben, consulting engineer, General Electric Co. This will be a general explanation of the American Welding Society's Structural Welding Code and a report on the progress of adoption of structural welding by cities and towns throughout the United States. "Progress of Design and Application of Structural Welding," by G. D. Fish, consulting structural engineer, Westinghouse Elec. & Mfg. Co. This is an illustration of the application of the Structural Welding Code to a hypothetical multistory building assumed to be erected within municipal building code jurisdiction, together with a report on accrued national welded tonnage in buildings and bridges. "Illustrated Review of Applied Structural Welding." Motion pictures and lantern slides showing applications of structural welding in buildings and bridges throughout the United States will be presented by American Bridge Co., Lincoln Electric Co., General Electric Co., Westinghouse Elec. & Mfg. Co., The Linde Air Products Co. and others.

"Progress in Civic and Professional Appreciation of Structural Welding." Several authorities prominent in civic and professional activities in and about New York City will discuss structural welding as it affects modernization of building construction methods and the comfort of those who must live or work in proximity to building operations.

Philadelphia

Planning, specifying, supervising and welding a 14-story office building in Boston will be the subject of the Nov. 17 meeting. J. T. Whitney, president, Whidden-Beekman Engineering Co., of Boston, will talk on planning, specifying and supervising. J. L. Faden, industrial engineer, The Edison Electric Illuminating Co. of Boston, will show motion pictures. A. G. Leake, president, Leake & Nelson Co., will have welding as his subject. This promises to be a very interesting meeting.

Pittsburgh

The first fall meeting of the Pittsburgh Section was held Wednesday evening, Oct. 22, in the Union Club. A paper on "Oxy-Acetylene Welding and Cutting in the Manufacture of Gas and Diesel Engines," prepared by W. F. Buchanan, foreman of welders for the Cooper-Bessemer Corp., was read in his absence by E. J. W. Eggers, of the Linde Air Products Co. A second paper on the "Maintenance Welding in the Steel Plant," prepared and presented by E. L. Quinn, Jr., supervisor of welding, Jones & Laughlin Steel Corp., was ably handled and proved conclusively Mr. Quinn's thorough familiarity with his subject. Quite a lively discussion ensued after the papers were read, and both Mr. Eggers and Mr. Quinn were kept busy answering questions.

The Pittsburgh Section is preparing a tentative program for the next seven months which will be published in the JOURNAL as soon as available.

Portland

At the annual meeting of the Portland Section the following officers were elected: Ray I. Smythe, chairman, Willamette Iron & Steel Co., Portland; G. C. Dierking, vice-chairman; W. P. Kellogg, treasurer; G. B. Herington, secretary, with offices in the Multnomah Hotel lobby in connection with duties as executive secretary, Associated General Contractors of America.

San Francisco

An illustrated lecture on the latest developments and applications of the Electronic Tornado, carbon arc method of welding was presented by Northern California representatives of the Lincoln Electric Co. at the Oct. 17 meeting of the San Francisco Section, held at the Athens Athletic Club, Oakland. This process welds 60 ft. of ⅜-in. plate per hour, with a homogeneous weld. New and important applications of fusion welding were one of the main attractions of this meeting.

Western New York

The following is a schedule of coming meetings of this Section:

November 10, 1930

Speaker: Mr. D. H. Deyoe, Industrial Eng. Dept., General Electric Company.

Subject: Structural Steel Welding.

Featuring (with movies and slides):

(a) Fourteen-story office building—Southern California Edison Company.

(b) Thirteen-story office building—Boston Edison Company.
(c) Nineteen-story office building—Dallas Light & Power Company.
(d) Battle deck steel floor. Welding—Berkshire Garage.
(e) Brief Outline of Recent Developments and Their Applications in Electric Arc Welding in General.

December 8, 1930

Speaker: Mr. W. R. Ost, Air Reduction Sales Company.

Subject: Transmission Pipe Line Welding (Oxy-Acetylene). Illustrated with movie and lantern slides.

February 9, 1931

Speaker: Mr. R. D. Thomas, vice-president, Arcos, Inc.

Subject: Arc Welding of Corrosion Resistant Steels. Illustrated with lantern slides.

March 9, 1931

Speaker: Mr. G. H. Koch, Westinghouse Elec. & Mfg. Co.

Subject: Automatic Welding. Illustrated with lantern slides.

Portland

The meeting of the Section was held on Oct. 30 at the Multnomah Hotel. A number of important items of business were transacted. Mr. F. Huggins, Superintendent Engineer of the Air Reduction Company, presented an interesting paper on "Progress of the Oxy-Acetylene Process in Railroad Shops," which described railroad welding practice and economies.

EMPLOYMENT SERVICE BULLETIN

Opportunities.—The Society is glad to learn of desirable opportunities from responsible sources, announcements of which will be published without charge in the BULLETIN.

Services Available.—Under this heading brief announcements (not more than seventy-five words in length) will be published without charge to members. Announcements will not be repeated except upon request received after an interval of three months; during this period, names and records will remain in the office reference files.

Note.—Copy for publication in the BULLETIN should reach the Society's Office not later than the thirtieth of the month if publication in the following issue is desired. *ALL REPLIES should be addressed to the number indicated in each case and mailed to the Society headquarters.*

SERVICES AVAILABLE

A-109. Electric welder desires position. Nine years' experience in job shop, car shop, steel mill and tank works. High school graduate. Location no object. U. S. A., South America, or any foreign country.

A-110. Welder desires position. Nine years' experience in welding in the Consolidated Aircraft, Buffalo.

A-111. Electric arc welder. Took course at Westinghouse Electric & Manufacturing Company.

American Methods of Welding Steel Structures*

BY FRANK P. McKIBBEN,†

IN a very short period of time, welding as applied to fabrication of new steel buildings and to strengthening existing bridges has advanced from the experimental to the practical stage. Only four years ago the safety and practicability of applying welding to steel buildings and bridges were questioned by most structural engineers and steel fabricators, but today much of this scepticism is dissipated, and while the process is by no means universally acceptable, the old inertia has quite largely been overcome and welding is progressing to the extent that nearly all structural shops are using the process to a greater or less extent. It must not be inferred from this that welding has replaced other forms of connecting steel members, for such is not the case; riveting remains the principal form of construction, but the usefulness of welding is fast being acknowledged.

The elimination of noise resulting from welding has become widely known, and in this lies welding's greatest asset at present; but a close second is its adaptability to repairing or strengthening old steel bridges or buildings where in most cases great economies result from its use. Some of the largest railroad systems have applied welding very successfully to strengthening steel girders or truss members in bridges which support tracks as well as in structures carrying highways overhead. And two railroad bridges have been completely welded.

The Present Status

Among the more important buildings being totally or partially welded in the early part of the year 1930 are the following:

Southern California Edison Company's office building approximately 150 ft. in height in Los Angeles, Cal., wherein the trusses and other earthquake resisting members are welded. Other portions of the steel frame are riveted.

Edison Electric Illuminating Company's office building, fourteen stories in height, in Boston, Mass., in which the steel is shop riveted and field welded.

The City Hall, Schenectady, N. Y., is completely shop and field welded.

An eleven story engineering laboratory is being completely welded in East Pittsburgh, Pa.

A four story factory building is being completely welded at Bridgeport, Conn.

A factory building at Oklahoma City, Okla., 140 ft. wide, 260 ft. long and 45 ft. high is being completely welded.

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*Presented at International Congress of Steel Construction, Liège, Belgium. Approved by Executive Committee, A. W. S.

†Consulting engineer: Senior vice-president, American Welding Society.

Steel building frames may be welded either with oxyacetylene or electric arc. The largest structure thus far welded with oxyacetylene is a research laboratory, 75 ft. wide, 260 ft. long and 42½ ft. in height, at Niagara Falls, N. Y., and although gas is widely used in welding other steel forms it has not been applied to any appreciable number of steel building frames, nor have atomic hydrogen, the shielded arc, or thermit welding been used for this purpose but are successfully used in many other places.

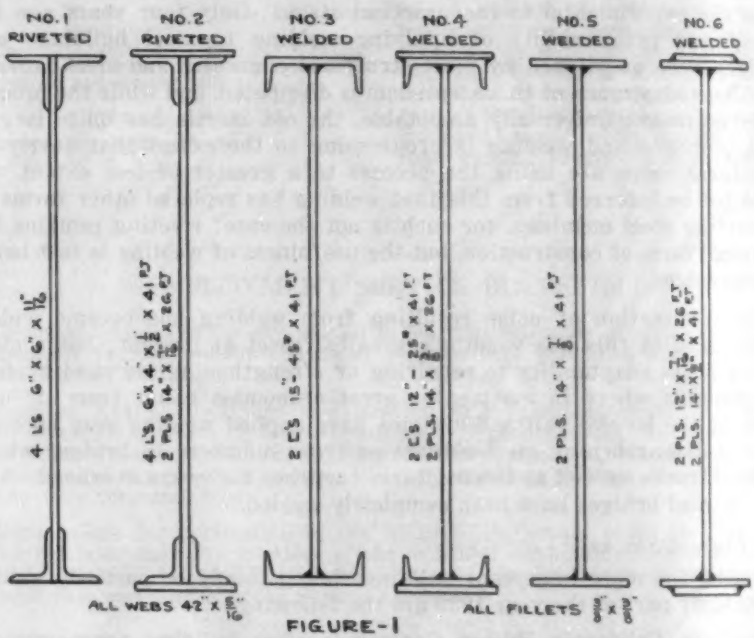


FIGURE-1

CENTRAL SECTIONS OF TWO RIVETED AND FOUR WELDED GIRDERS

Nearly all welding on steel building frames has been performed with the metallic arc. However, almost without exception those cities which have modified their building codes to permit welding have included both gas and electric welding, as they should.

Electric resistance processes are being applied on a large scale to the spot and butt welding of bar joist and other similar building parts. These processes seem to be quite suitable for shop fabrication of light material.

In the United States over a hundred steel buildings have been totally or partially welded. No failure has occurred in any of the welded joints.

Machine Welding

The past year has witnessed an interesting extension in the use of automatic gas or electric welding machines for shop fabrication of structural steel and for steel pipe manufacture. Ordinary welded connections of beams to beams or beams to columns do not lend themselves to machine

welding, but in shop fabrication of plate girders or composite columns having long lines of welds automatic welding is being increasingly applied with success and economy. Among the interesting installations of this kind are those of the Mississippi Valley Structural Company at Decatur, Ill., where two arc welding heads may be used simultaneously for such operations as welding a plate girder web to a flange plate with two parallel fillet welds; and of the Cleveland (Ohio) Crane Company, where two welding heads perform a similar function on traveling crane box girders, welding the two webs to a flange plate.

Economies

Aside from the economic value of noise elimination in many locations, the savings resulting from welding lie principally in the use of less steel in trusses or plate girders. In multi-story building frames consisting of welded beams and columns the decrease in weight of steel is small unless the beams be made continuous, in which case the bending moments are

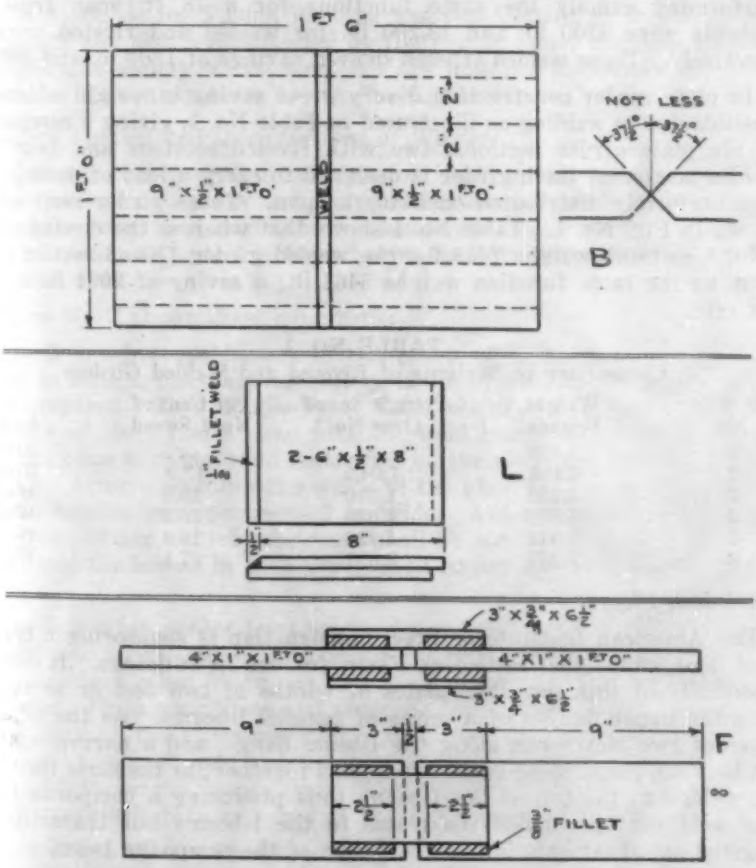


FIG. 2—SPECIMENS USED IN TESTING QUALITY OF WELDS

sufficiently diminished to permit lighter beams; but thus far the cost of additional welding necessary to acquire continuity has offset the saving in steel so that in multi-story buildings welding has not shown any decided economic values over other connections. However, in two office buildings, each fourteen stories high, now being built, the cost to the owners of the steel frame shop riveted and field welded is in each case the same as if both shop and field connections were riveted. This equality in cost reveals a decided advance in the use of welding in a very short time. But it is too early to predict the comparative costs of riveted and welded multi-story steel frames, because no single fabricating company has had sufficient tonnage of welded work to ascertain cost data under normal production.

In ordinary roof truss construction such as that applicable to mill buildings very considerable savings in weight of steel are possible; for example, for roof trusses having spans of 58 ft. 6 in. the weight of each welded truss was 5000 lb. as compared with 6800 lb. for a riveted truss performing exactly the same function; for a 78 ft. span truss the weights were 9200 lb. and 13,200 lb. for welded and riveted work respectively. These welded trusses showed savings of 1800 lb. and 4000 lb.

In plate girder construction a very great saving in weight of steel is possible due to welding as illustrated in Table No. 1, giving a comparison of six plate girder sections, two with riveted sections and four with welded sections. Each girder is designed to carry a load of 4000 lb. per foot uniformly distributed on a 40 ft. span. These girder sections are shown in Fig. No. 1. Table No. 1 shows that whereas the riveted girder (No. 1 section) weighs 7498 lb., the welded girder (No. 6 section) performing the same function weighs 5467 lb., a saving of 2031 lb. or 27.1 per cent.

TABLE NO. 1

Comparison of Weights of Riveted and Welded Girders

Section No.	Weight, Pounds	Pounds Saved Over No. 1	Per Cent of No. 1 Saved	Remarks
1	7,498	....	....	Riveted
2	6,455	1,043	13.9	Riveted
3	6,282	1,216	16.3	Welded
4	5,572	1,926	25.7	Welded
5	6,012	1,486	19.8	Welded
6	5,467	2,031	27.1	Welded

Battledeck Floors

The American Institute of Steel Construction is sponsoring a type of steel floor, called the Battledeck Floor, for use in buildings. It consists essentially of thin flat steel plates of widths of two feet or so resting upon the upper flanges of a series of parallel I-beams. As the adjacent edges of two plates run along the I-beam flange, and a narrow space is left between them, these edges are welded together; at the same time both are welded to the top of the I-beam, thus producing a composite beam. The weld not only holds the plates to the I-beams but transmits the longitudinal shear existing in the flange of the composite beam.

To construct this floor an automatic self propelled welding machine is

available, which at a demonstration before the Institute of Steel Construction at its annual convention in 1929 welded the seams at an average rate of 9½ in. per minute on 3/16 in. steel floor using 275 amperes and 22 volts in the arc with a 3/16 in. diam. electrode.

Welding and the Building Codes

Practically all cities and many towns in the United States have building codes which to a greater or lesser extent regulate the types of buildings and the materials of which they are constructed. During the past two years, 77 towns and cities have amended their building codes to incorporate permission to use fusion welding in steel building frames. The maximum permissible unit stresses, specified in all of these codes for proportioning welded joints under quiescent loads are as follows:

Shear on section through throat of weld.....11,300 lbs. per square inch
Tension on section through throat of weld.....13,000 lbs. per square inch
Compression on section through throat of weld....15,000 lbs. per square inch

The above are the units recommended by the American Welding Society in its "Code for Fusion Welding and Gas Cutting in Building Construction"; and also recommended by the Pacific Coast Building Officials Conference, a society whose members are building inspectors and commissioners of municipalities on the Pacific Coast.

Procedure Control and Qualifications of Welders

Procedure control, or the technique of welding, has greatly improved during the last two years and the qualifications of welders to perform their tasks have been particularly emphasized. The writer uses three types of specimens to test welders before they are permitted to weld any important work.

Figure No. 2 shows these specimens:

Type B is a butt weld made of two pieces of 9 in. by ½ in. by 12 in. welded to form a piece approximately 12 in. by 18 in.; the welded edges being prepared with a double-Vee. Each welder is required to make at least two of these plates; one with the weld horizontal, i. e., the plates lying flat; one with the weld vertical, i. e., the plates on edge during the welding. After machining the welds to the plate thickness each plate is cut into tension test specimens 2 in. wide. Average tensile strength of each group of arc welded specimens shall be not less than 45,000 lb. per sq. in., and the lowest in each group shall be not less than 40,000 lb. per sq. in.

Type L is a lap welded joint made of two 6 in. by ½ in. by 8 in. plates clamped one on the other with the 8 in. edges in alignment, but with the 6 in. edges offset ½ in. A full ½ in. fillet weld is made along one 6 in. edge. Upon completion and after cooling the plates are broken apart by wedging at the unwelded 6-in. edges, and the fractural metal shall show good fusion of weld and base metals; good penetration into the root of the fillet; bright, dense, even textured, crystalline or fibrous weld metal, irregularly torn and void of iridescent colors.

Type F is a fillet welded specimen consisting of two 4 in. by 1 in. by 12 in. plates spliced for a tension test with two 3 in. by ¾ in. by 6½ in.

splice plates. The welds are ⅜ in. triangular in section and each 2½ in. long, making 10 in. of fillet on each side of the splice. The craters in this specimen are filled to the full section of the weld. Each welder is required to make at least two of these specimens which, when tested in tension, shall show at least an average ultimate longitudinal shearing stress of 44,000 lb. per sq. in., and the lowest in the group shall show at least 38,000 lb. per sq. in. of minimum section of fillet.

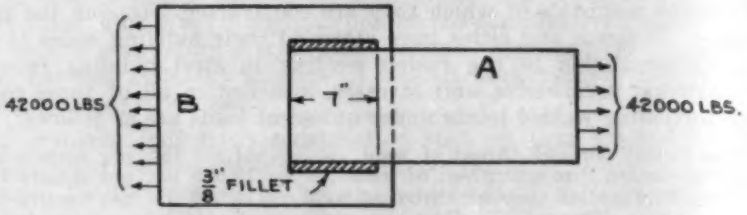


FIG. 3—FILLET WELDS IN LONGITUDINAL SHEAR

Table No. 2 exhibits a series of qualification tests to determine the ability of three welders. It is noted that the ultimate tensile strength of the welded joints as determined by tension tests on 2 in. by ½ in. strips was from 49,000 to 55,700 lb. per sq. in.

Table No. 3 sets forth a series of qualification tests to determine the ability of three welders whose fillet welded specimens like F in Figure No. 2 gave ultimate longitudinal breaking strengths of from 133,100 lb. to 155,200 lb. corresponding to ultimate longitudinal shearing strengths of 13,310 and 15,520 lbs. per linear inch for ⅜ in. fillets.

The test data of Tables No. 2 and No. 3 are typical of welding of this sort, of which there are sufficient data for the design of all ordinary cases; but when the American Bureau of Welding's Committee on Structural Steel Welding completes its very comprehensive series of tests on gas and on electric welded joints a splendid lot of data will be available.

Strengthening Steel Structures by Welding

A very interesting development is the application of welding to strengthening existing steel bridges and buildings which are overstressed.

Some outstanding examples of bridge strengthening are:

Reinforcing an old railroad bridge over the Missouri River at Leavenworth, Kan., consisting of one 400 ft. movable draw span and two 300 ft. mixed spans of steel trusses, which on account of increases in weight of rolling stock had become too light for recent traffic. In this work 225,000 lb. of structural steel were added by welding; 7500 linear feet of welds and 2000 lb. of welding wire were used during the reinforcement which took place between Feb. 28 and April 28, 1928, without interrupting normal railroad traffic.

An old single track Pennsylvania Railroad Bridge over Susquehanna River at Havre de Grace, Md., had an additional floor system welded into the old trusses above the existing floor to convert the structure into a

double-deck highway bridge for heavy automobile traffic. This bridge, consisting of several spans, is about 3000 ft. long; 1500 tons of steel were thus added to this structure by the Maryland State Road Commission.

In some cases, in strengthening existing trusses or girders by welding additional steel to the old work, a transference of a part of the dead load stress from old to new work is necessary when, without such transfer, the dead load stress in old work would remain too large.

TABLE NO. 2

Results of Qualification Tests of Three Welders.

Tension Tests on Butt Welds. See Specimen B, Figure No. 2. Minimum cross section of each specimen approximately 2 in. by 1/2 in.

Welder Number	Specimen	Area of Cross Section, Sq. In.	Tensile Strength, Lb. per Sq. In.	Fracture
A	B1	0.88	53,400	Outside of weld
A	B1	0.87	50,400	Outside of weld
A	B1	0.88	53,800	Outside of weld
A	B1	0.88	54,200	Outside of weld
A	B1	0.88	54,400	Outside of weld
A	B2	0.88	54,600	Outside of weld
A	B2	0.87	55,000	Outside of weld
A	B2	0.87	54,800	At weld
A	B2	0.90	54,600	Outside of weld
A	B2	0.86	55,700	Outside of weld
B	B1	0.94	54,500	At weld
B	B1	0.94	55,000	At weld
B	B1	0.98	54,000	At weld
B	B1	0.98	54,600	At weld
B	B1	0.98	53,800	Outside of weld
B	B2	0.98	50,600	At weld
B	B2	0.97	51,200	At weld
B	B2	0.97	52,200	At weld
B	B2	1.00	53,400	At weld
B	B2	0.99	49,000	At weld
C	B1	0.88	52,600	At weld
C	B1	0.89	55,200	Outside of weld
C	B1	0.89	52,800	At weld
C	B1	0.86	50,600	At weld
C	B1	0.85	52,000	At weld
C	B2	0.88	52,800	At weld
C	B2	0.88	50,200	At weld
C	B2	0.90	50,800	At weld
C	B2	0.87	52,700	At weld
C	B2	0.89	53,400	Outside of weld

TABLE NO. 3

Results of Qualification Tests of Three Welders. 3/8 In. Fillet Welds Tested in Longitudinal Shear. See Specimen F, Figure No. 2.

Welder Number	Yield Point, Lbs.	Ultimate Loads, Lbs.	Ultimate Load, Lbs. per Linear Inch of Fillet
A	117,000	149,000	14,900
A	116,000	154,000	15,400
B	119,000	133,600	13,360
B	117,500	133,100	13,310
C	115,500	140,500	14,050
C	114,900	155,200	15,520

Consider, for example, a plate girder tension flange consisting of two angles to which is to be welded a plate to increase the flange area. If the plate be welded without preheating, the dead load stress remains carried by the angles only. To transfer a part of the stress to the plate, the girder must needs be temporarily supported and raised during the welding, or else the plate must be preheated to a proper temperature so that when welded it will carry its full share of dead load stress. The problem can be shown thus:

1. How many degrees, D, increase above air temperature are required to cause a unit stress, in the added steel of S lb. per square inch?

Change in length of added steel due to stress $= \frac{SL}{E}$

Change in length of added steel due to heat $= DCL$

Equating these and solving for D, $D = \frac{S}{EC}$

Where S = unit stress in the steel added.
L = length in inches of steel added.
D = degrees Fahrenheit.
C = linear coefficient of expansion per degree Fahr.
E = Young's modulus of elasticity.

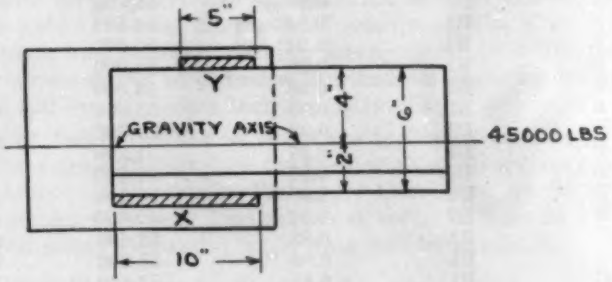


FIGURE-4

DISTRIBUTION OF WELDS WITH MEMBERS OF UNSYMETRICAL CROSS SECTION.

2. How may the above change in temperature with corresponding unit stress be most simply determined and secured?

By welding to the old steel one end of the new steel and heating the new steel till its length increases an amount equal to DCL; then while the new steel is hot weld the other end, and complete the intermediate welding.

For example: Suppose it is desired to cause a stress of 10,000 lb. per square inch in an added plate having a length of 300 inches, how many

degrees Fahrenheit must the plate be heated and what will be the corresponding increase in length?

$$D = \frac{S}{EC} = \frac{10000}{30000000(0.000007)} = 50 \text{ deg. Fahr.}$$

$$\text{Increase in length} = DCL = 50(0.000007)300 = \frac{1 \text{ in.}}{10}$$

$$\text{or more directly, increase} = \frac{SL}{E} = \frac{10000\ (300)}{30000000} = 0.1 \text{ in.}$$

The above method was applied to strengthening a New York Central Railroad over head bridge in the Bronx, New York, between East 157th Street and East 159th Street.

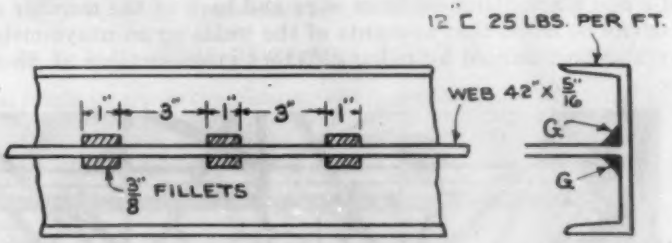


FIGURE-5

PLAN OF LOWER FLANGE OF PLATE GIRDER SHOWING REQUIRED SPACING FOR 3/8-INCH FILLETS, CONNECTING WEB AND FLANGE.

Design of Welded Joints

In designing welded joints the same general principles are to be followed as in designs of riveted construction—gravity axes of truss members should meet at a point or allowance made for the eccentricity; welds should be distributed in accordance with the location of the gravity axes. In general, the practice is to so arrange joints in trusses to throw fillet welds in longitudinal shear. However, as it is often desirable to use them in transverse shear, recent tests indicate that the ultimate shearing strength of these two classes are nearly the same.

An example of a joint with fillet welds in the longitudinal shear is shown in Fig. No. 3, where a stress of 42,000 lb. is to be transferred from plate A to plate B; using $\frac{3}{8}$ in. triangular fillets at 11,300 lb. per sq. in. The throat or minimum dimension of the weld is 0.266 in., which, at 11,300 lb. per sq. in., is 3000 lb. per linear inch of fillet. Fourteen inches of fillet are required, 7 in. on each side, as shown in Fig. No. 3.

But if the tension piece A be of unsymmetrical cross section, the distribution of the welds would be as shown in Fig. No. 4; the total number

of linear inches of fillet at 3000 lb. per linear inch being 15 in., arranged as in Fig. No. 4, with 10 in. at X and 5 in. at Y.

$$\text{Fillet at X} = \frac{15(4)}{6} = 10 \text{ in.}$$

$$\text{Fillet Y} = \frac{15(2)}{6} = 5 \text{ in.}$$

or, in general, $$\text{Fillet at X} = \frac{P\ b}{S(a+b)}$$

$$\text{Fillet at Y} = \frac{P\ a}{S(a+b)}$$

Where S is the permissible unit strength of fillet in lb. per linear inch, and a and b are distances from edge and back of the member respectively. It should be noted that moments of the welds on an unsymmetrical section may also be balanced by using different cross-sections of fillet welds.

FIG. 6—GENERAL ELECTRIC CO.'S ARC WELD BUILDING AT WEST PHILADELPHIA, PA.

Another example of the use of fillet welds is shown in Fig. 5, where the flange, consisting of a channel, is welded to a web of a plate girder by intermittent welding. Given a web 42 x 5/16 in.; a flange of one 12-in. channel weighing 25 lb. per ft. with its center of gravity 0.68 in. from its back, a web thickness of 0.53 in., and an area of 7.35 sq. in.; an end vertical shear on the girder of 80,000 lb. resulting from a load of 4000 lb. per linear foot on a span of 40 ft. Assuming 3/8 in. intermittent fillets each 1 in. long connecting the 12 in. channel to the 42 x 5/16 in. web, how far apart in the clear should these fillets be spaced to carry the longitudinal shear from flange to web? Use 3000 lb. per linear inch for the fillet 1 in. long.

At end of girder, the longitudinal shear is $\frac{80{,}000 \times 153}{8338} = 1470$ lb. per linear inch for two fillets marked G in Fig. 5, or 735 lb. per linear inch for one side.

The 153 is the statical moment of one channel with respect to neutral axis of the girder and $= 7.35(21.53-0.68) = 153$. The 8338 is the moment of inertia about the girder's neutral axis and is found thus:

$$\frac{1}{12} \times \frac{5}{16} \times (42)^3 = 1929$$

$$2 \times 7.35(21.53-0.68)^2 + 4.5 = \frac{6409}{8339} \text{ inch units}$$

The spacing required for the ⅜ in. x 1 in. fillets at the end of the girder is, therefore, $\frac{3000}{735} = 4$ in. approximately, arranged as in Fig. 5. This requires ⅜ in. x 1 in. fillets alternating with 3 in. open spaces. If ⅜ in. x 2 in. fillets be used, the clear spacing would be, $\frac{3000 \times 2}{735} = 8$ in., requiring 2 in. fillets alternating with 6 in. open spaces on each side of the web. Although this computation has been made here to illustrate the method, most designers use continuous ⅜ in. fillets, not intermittent, for such cases.

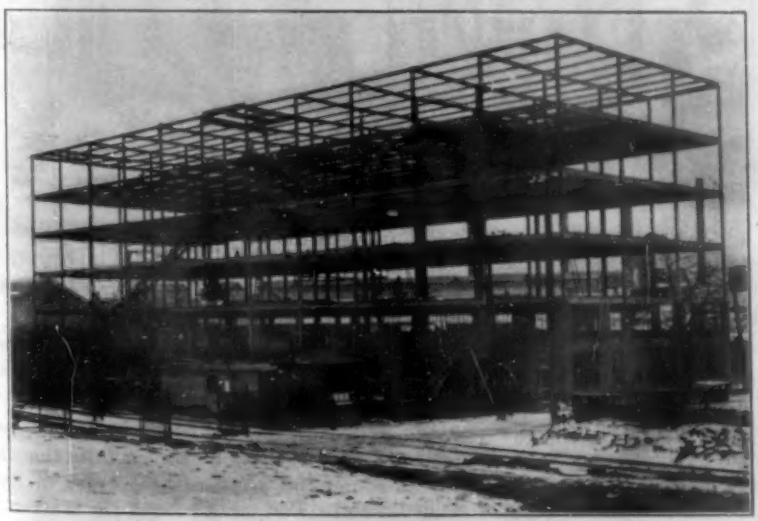


FIG. 7—WESTINGHOUSE ELEC. AND MFG. CO.'S WELDED BUILDING AT SHARON, PA.

Testing the Quality of Welds

Testing welded specimens to destruction gives data for use in designing welded joints, affords a satisfactory method of qualifying welders before beginning to weld an important structure and aids in maintaining a standard of workmanship during the period of construction. But destructive tests on the structure itself to ascertain the quality of welding

are not, in general, practicable. Consequently, attempts are being made to secure a non-destructive test by which the work can be tested and its safe carrying power assured.

Visual inspection still remains as the method most extensively used; in fact, so far as buildings and bridges are concerned, this is practically the only means employed. That it has been fairly satisfactory is shown by the absence of any failures in the welded buildings thus far erected, some of which have received very careful visual inspection and others very little

FIG. 8—LINDE AIR PRODUCTS CO.'S OXY-ACETYLENE WELDED BUILDING AT NIAGARA FALLS, N. Y.

inspection of any kind, a state of affairs common in all types of building construction.

Nevertheless, considerable progress has been made in devising non-destructive tests for welds. Among the most widely discussed methods are:

Electrical; by which the resistance to passage of current is utilized to detect the location and magnitude of a defect, such as a crack, within the weld.

Stethoscope; to detect by means of sound the presence of a crack or other spot differing from the normal welded metal. The main tube of the instrument is held on any part of the welded specimen while the weld is struck by a hammer.

X-ray, whereby a weld is photographed to locate defects. This method has not yet arrived at a stage where it can be utilized in a practical manner for testing building construction but as a means of analyzing the quality of work done in order to establish a proper welding technique it is of merit.

All of these non-destructive tests are of value, but none have been used to any large extent for investigating the quality of welds in steel bridges or building frames.

Arc Welding for High Structures*

By Gilbert D. Fish†

WELDING for building construction is receiving considerable attention in connection with proposed modification of building laws in New York and some other cities. The interest of the public, as reflected by the press, is concerned chiefly with the noise-eliminating feature of the welding process rather than with cost-saving or technical advantages.

Many cities and towns have already permitted welded building construction, in most cases without any changes in or additions to their building ordinances. Important instances can be cited in Newark, Boston, Philadelphia, Atlantic City, New Haven, Cleveland, Dallas, Los Angeles and Albany, but most if not all of the welded or partly welded buildings in those cities have been regarded by the authorities as special examples which did not commit them to recognizing the welding method for buildings generally. The largest all-welded building is the new 11-story Westinghouse laboratory at East Pittsburgh, where no ordinances govern construction methods, and several other record-making operations have been carried out in small towns where there has been no opportunity to establish valuable legal precedent.

It is noteworthy that welded building practice has grown and set a fine record of success in the face of serious interference by building ordinances in our largest cities. This obstruction has retarded the use of welding for commercial buildings more than for industrial buildings and has had a great deal to do with the low average height of welded structures. The first large welded structure of any kind was the Westinghouse factory at Sharon, built 4 years ago; it was 80 feet high. The Sharon building was not approached in height by any other welded structure until the Yale Library Book Tower, 135 feet high, and the 13-story addition to the Homestead Hotel at Hot Springs, were erected in 1928. The new Boston Edison building, shop riveted and field welded, is 15 stories; the recently erected Edison building in Los Angeles, mainly of welded construction but containing some shop rivets, is 12 stories.

At a time when competition in high building construction is bringing into existence office buildings taller than the historic Eiffel Tower, it would be surprising if welded structure of 12 to 18 stories should greatly impress the public, even though thoughtful engineers realize that any limitation of the welding process making it unsuitable for tall structures would have become apparent in the fairly high buildings already constructed. It is precisely because welded structures of great height have not yet been built that certain tremendous advantages of welding over riveting for that class of work have not yet been realized in practice or recognized by more than a few engineers. Quite recently, in connection with proposed modifications of the New York City building code, there

*Presented before the New York Section of the A.W.S., Sept. 16, 1930.
†Consulting Structural Engineer, Westinghouse Electric & Mfg. Co.

appeared in the press an expression of opinion, attributed to a group of engineers, to the effect that experience and precedent justify the use of welding in "structures of medium height where no wind stresses are involved." This reservation seems to me unfortunate and misleading, because tests and practice have proved the superiority of welded joints over riveted ones for resisting the kinds of stress caused by wind. There is evidently an immediate need for stating in the plainest possible terms the reasons why welding possesses outstanding advantages for wind bracing, and why welded construction can definitely be pronounced suitable for buildings of extreme height.

Tall buildings are distinguished from low ones by heavy columns and by bracing designed to resist lateral pressure from high winds. The upper half of a 30-story building is, in the structural sense, a 15-story building and is designed as such; the lower half is very different from a 15-story building because of the weight of the stories above with their contents and because of the wind pressure which must be transmitted from the upper stories to the foundation.

If architectural considerations did not forbid, wind bracing for very tall or very slender buildings would generally be accomplished by placing X-bracing in some of the vertical panels formed by columns and floor beams, thus forming vertical trusses capable of transmitting the horizontal wind forces to the foundation without inducing bending stresses in the members or connections. As the X-bracing method can but rarely be used, wind bracing is usually provided by making the beam-to-column joints strong enough in bending resistance to prevent collapse of the rectangular panels. Although there are numerous conflicting theories as to the distribution of wind bracing stress among the many joints in any one floor of a building, it is usually nearly true that the aggregate of wind bending moments in the joints of a given floor equals the total wind pressure above that floor, multiplied by the story height. A building does not need to be uncommonly high to have serious bending stresses in the joints of its lower stories.

The commonest way to provide bending resistance in a riveted joint connecting a beam to a column is to connect both top and bottom flanges to the column by means of angles or other connecting pieces riveted to the flanges of the beam and to the column. Bending stresses due to wind are resisted by a pair of horizontal forces at top and bottom connections, one force developing tension between beam flange and column and the other force compression. These forces alternate with reversal of wind direction, so that both connections act sometimes in tension and sometimes in compression. The limit of bending resistance of such a joint is generally governed by the strength of top or bottom attachment in tension, which tends to pull the connecting member out of shape and elongate the rivets of the column connection. Sometimes, when the connection is very heavy and contains many rivets, the limit of resistance is governed by the tensile strength of the beam flange, weakened by rivet holes. Triangular gusset plates or diagonal knee-braces are occasionally employed as reinforcement to increase the strength and stiffness; they are expensive and architecturally objectionable in most cases.

Any one of several forms of welded beam-to-column connections can be made to develop any required bending resistance, up to the limit of

strength of the main members, which are not weakened by rivet holes. Such a connection is simple, compact and reasonable in cost; in no case does it require projecting gusset plates or knee braces, unless for the purpose of reducing the stresses or the deformations in the main members. One photograph of such a connection is sufficient to illustrate the general fact. Such a joint has no element which can work loose or permit high local strains such as are common in riveted connections used for the same purposes; it follows that a building frame, braced by properly designed welded beam-to-column connections, cannot drift due to play or flexibility in the joints; all of the drift or deflection of such a frame is due to elastic

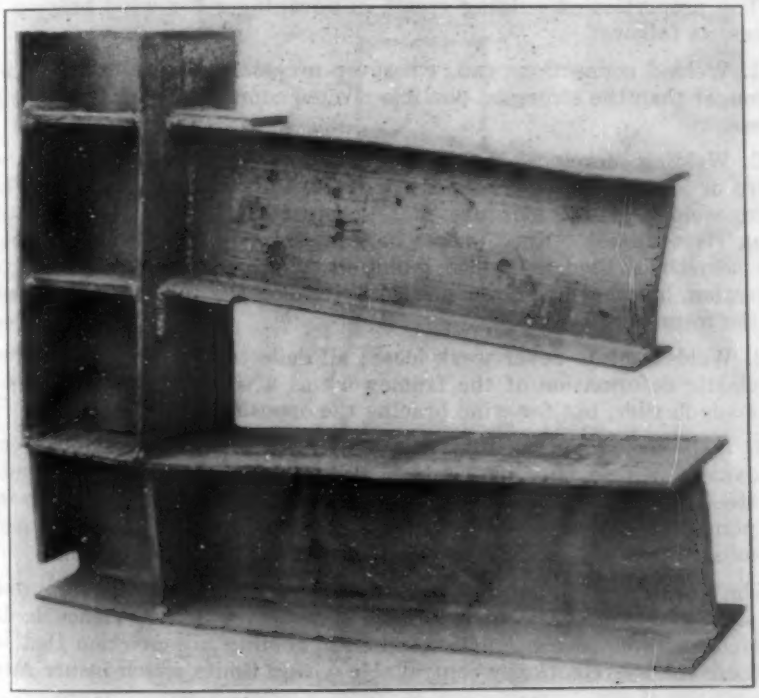


FIG. 1

deformation of the framework as a whole. For the sake of comfort of occupants of the upper stores of very tall buildings it is sometimes necessary to confine the drift caused by wind within narrower limits than are imposed by strength requirements alone. Wherever stiffness rather than strength is the criterion for wind bracing design, welding has the advantage of greater stiffness than riveting. The taller the structure the stronger the reasons for preferring welded wind bracing.

For some classes of joints commonly occurring in building frames, flexibility is necessary to avoid dangerous secondary stresses, and in such cases welded joints should be designed for flexibility of the same order as is found in riveted joints used for the same purposes. Wind bracing connections do not fall under this heading; they should be made rigid,

and no drawback of any sort results from making them so. Beams connected to columns by very rigid wind connections, whether welded or riveted, are fixed or continuous beams and should be designed as such to carry floor loads and wind load combined. The prevailing American practice of treating riveted beams as simple or unrestrained in their action under floor loads leads to serious inconsistencies in design when the connections are made very rigid to resist wind. Whereas ignoring partial restraint at end connections results in error on the side of safety in designing beams for floor load only, the error is on the danger side in designing beams to resist wind bending and floor loads at the same time.

The properties of welding result in advantages for wind bracing purposes, as follows:

1. Welded connections can, wherever necessary or desirable, be made stronger than the strongest possible riveted connections for the same purposes.

2. Welding develops highly concentrated resistance, requiring one-third or less of the space needed for rivets to transfer equal load. Therefore, welded connections are more compact and require less material than riveted ones. Knee braces or gussets projecting below the beams are sometimes used in welded buildings where there is no architectural objection, but they are never needed to develop the full available strength of the members.

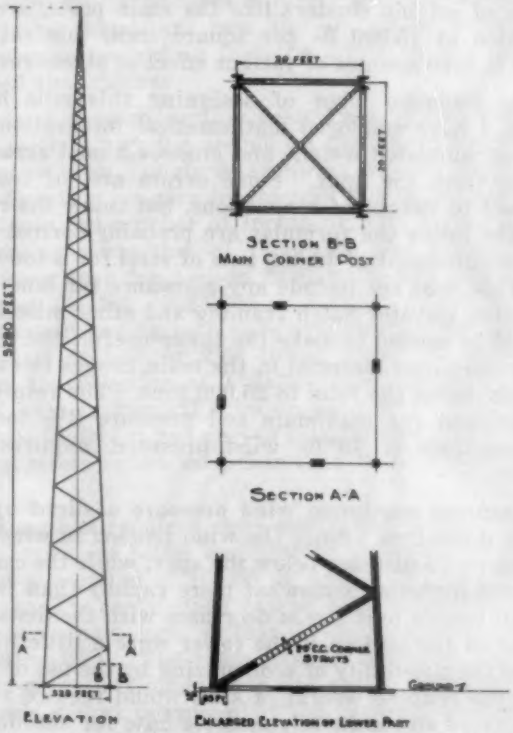
3. Welded joints never work loose; all deflection or drift is due solely to elastic deformation of the framework as a whole. Welded joints can be made flexible, but for wind bracing the opposite is needed.

4. It is easy to prove that field welding costs less than its strength equivalent in field riveting. Add to this saving the reduction in amount of steel required for a connection and the elimination of the shop work of punching or drilling rivet holes. The result is that it saves money to substitute a well-designed welded connection for a riveted one.

The question of reliability does not favor either riveting or welding as against the other. No one could have years of experience in both riveted and welded construction, without holding a conviction that both methods are systematically controllable within limits which insure safety.

Buildings are not the only tall structures. Until recently the Eiffel Tower, nearly 1000 feet high, held the world's record. Possibly some organization will presently decide to erect a far taller tower for scientific purposes or for service in connection with aviation or for advertising and public amusement. It goes without saying that a tower lacking solid walls and solid floors can be built far higher than any possible building structure, but I am not aware that any engineer has worked out figures as to the limiting heights as governed by strength and weight of available materials. With the idea of carrying the discussion of tall structures into dimensions which have not been explored by actual construction, I have made a rough study of an open steel tower one mile in height. While there may be no likelihood that such a structure will be built, brief consideration of its possibilities is repaid by the resulting knowledge that it is practicable and by certain items of information as to a feasible design and the amount of steel required for same.

The object of introducing this hypothetical tower into the present discussion is to point out the reasons why arc welding would be preferable to riveting for such a structure. A skeleton diagram of such a tower is shown, with items of information about the form and dimensions of a few of the main members. No comparative studies have been made to determine whether different proportions would result in greater economy, but the base width, taken at one-tenth the height, is evidently about as small as could be used under the assumed wind pressure without serious uplift at base of windward columns. For designing the steel, wind pressure



STUDY OF MILE-HIGH ARC-WELDED STEEL TOWER
THEORETICAL REQUIREMENT 15000 TONS OF STEEL

FIG. 2

has been assumed at 20 lbs. per square foot of exposed vertical surface, but as this figure has been applied to all vertical windward faces even where partially protected by neighboring members, the allowance represents more than 20 lb. pressure on fully exposed surfaces. Taking into account the low ratio of average to maximum pressure where the affected area is very extensive, and the fact that occasional winds of much higher average intensity than the assumed value could safely be resisted by the structure as designed, I think the 20 lb. figure is fair. The foundation has been tested for stability and soil pressure under 50 per cent extra wind pressure. Before designing such a high tower for actual construction, I should want to investigate the wind pressure more carefully, with

special reference to wind velocities at high altitudes. All members are designed for compression, but require connections and splices designed to resist both compression and tension. All main members, including the diagonal wind braces, are proportioned with length ratio equal to 60. The main posts, forming the four edges of the pyramid, are composed each of a cluster of four columns arranged in a square, braced by trusses in the four outer planes and by transverse bents; the main compression elements are designed for combined effect of the weight above and maximum wind compression, at 20,000 lb. per square inch. The main diagonals, made up of column clusters like the main posts, are designed for wind compression at 15,000 lb. per square inch, this rather low value being intended to take account of fatigue effect of stress reversal.

To avoid the immense labor of designing this mile high structure panel by panel, I have employed mathematical integration to determine wind stresses, accumulated weight and cross-sectional areas as functions of the distance from the apex. Some errors are, of course, involved, mainly in respect to weight of connections, but below the relatively light top fourth of the tower the formulas are probably correct within 10 per cent. I find a requirement of 19,000 tons of steel for a tower with welded connections. This does not include any allowance for landing stages, observation galleries, elevator hatch framing and other miscellaneous framing which would be needed to make the tower useful; the weight of these extras and the additional material in the main towers to carry the added load might easily bring the total to 25,000 tons. The reinforced concrete foundation, designed for maximum soil pressure 2½ tons per square foot, under conditions of 30 lb. wind pressure, requires 13,000 cubic yards of concrete.

Under the assumed maximum wind pressure a slight uplift exists at base of windward columns. Since the wind tension in windward columns varies as the square of distance below the apex, while the compression due to weight of steel increases somewhat more rapidly than the cube of the distance, the net tensile unit stress decreases with the distance down and would disappear at the bottom if the tower were a little higher. I have not investigated the possibility of economizing by the use of high-strength alloy steel, but the reduced weight of steel would require a wider spread of structure to avoid an excessively massive base for stabilization.

From consideration of the general character of connections required for main panel points, lacing truss joints and splices, I estimate that riveted construction would require additional connection material amounting to about 10 per cent of the total steel in the structure, and some additional material in the main columns to carry that additional weight. This difference in amount of steel is alone a substantial reason for preferring welding. Another reason is the simplification of fabrication due to saving rivet holes in very thick material; another is the general cost saving in the field welding operation as compared with driving equivalent rivets; still another is the reduced cost of repainting steel which is free of rivet heads. Finally, there is an indeterminate amount of extra stiffness in a welded structure due to complete absence of slip and play in the joints. If such a structure, or an even higher one, is ever built, I think it will be welded.

A Rational Method of Welded Connection Design

By Andrew Vogel†

ENGINEERS have become so accustomed to standard riveted connections of beams to girders or columns that the method of determining the number of rivets required is rarely considered. Welding connections, on the other hand, are new and should be subjected to careful analysis in order to be certain that the stress in any part of the weld does not exceed that desired.

Two methods of weld connection design are in use: the first is known as the shear method and the second as the shear-and-moment method. The shear method involves only determining the end reaction of the beam and then dividing this reaction by the unit stress per linear inch of weld in order to obtain the number of inches of weld metal required. As an example, assuming a reaction of 16,800 lb. and a unit stress value of 3000 lb. per linear inch for a ⅜-in. fillet weld, 5.36 linear inches of weld will be required (16,800 ÷ 24,000 = 5.36). Assuming that an angle bracket will be used having both ends available for welding, there will be provided one-half of 5.36 or 2.68 linear inches of weld on each edge. This indicates the selection of a 3x3x7/16-in. angle as the seat angle, with 3 in. of weld in vertical lines on each end. The shear method is short, and while to date it has proved satisfactory, this is more the result of the low unit stress values used than it is of the efficiency of the method itself.

The second method requires more study, but the results are more conservative and represent better engineering. Considering again the above assumption of a reaction of 16,800 lb. and ⅜-in. fillet welds at 3000 lb. per linear inch: The metal itself can be designed for a bearing pressure of 24,000 lb. per square inch, so that 0.7-sq. in. bearing is required (16,800 ÷ 24,000 = 0.7). The beam flanges, however, will deflect upward if this pressure is distributed over the entire width of flange, and the angle clip would deflect downward if the pressure is distributed over its width or concentrated at its outer edge. Analysis of the beam shows that a pressure of 24,000 lb. per square inch may be distributed over a width equal to the thickness of web plus the depth of flange where the flange joins web, without overstressing the lower fibers of the beam. A study of the angle indicates that, because of the relative rigidity of beam and angle leg, the leg will deflect; thus placing the actual center of the reaction near the fillet of the angle. Construction considerations also require that beams shall be cut short so that they can be readily placed and also that lengths may be received plus or minus ⅜ in. or mill tolerance. It will be assumed that a gap of ⅜ in. exists between the end of the beam and the surface of the column to which it is to be connected.

The calculation of the beam-bearing area proceeds from the dimensions

†General Electric Co., Schenectady, N. Y.

of the beam. Assume the beam in question to have a web thickness of 0.355 in. and a flange thickness at the surface of the web of 0.764 in. The sum of 0.355 in. and 0.764 in. is 1.12 in., one dimension of the bearing area. Since the bearing area required is 0.7 sq. in., the other bearing-area dimension is 0.7 ÷ 1.12, or 0.63 in. The bearing area, therefore, is 0.63 in. parallel to the beam axis and 1.12 in. perpendicular to the beam axis, as shown in the accompanying illustration. Adding 0.63 in./2 to 0.375 in. equals 0.69 in., the moment arm. As the reaction is 16,800 lb. the moment at the surface of the column is 16,800 × 0.69 in., or 11,600 in.-lb. The shear at the surface of the column is 16,800 lb.

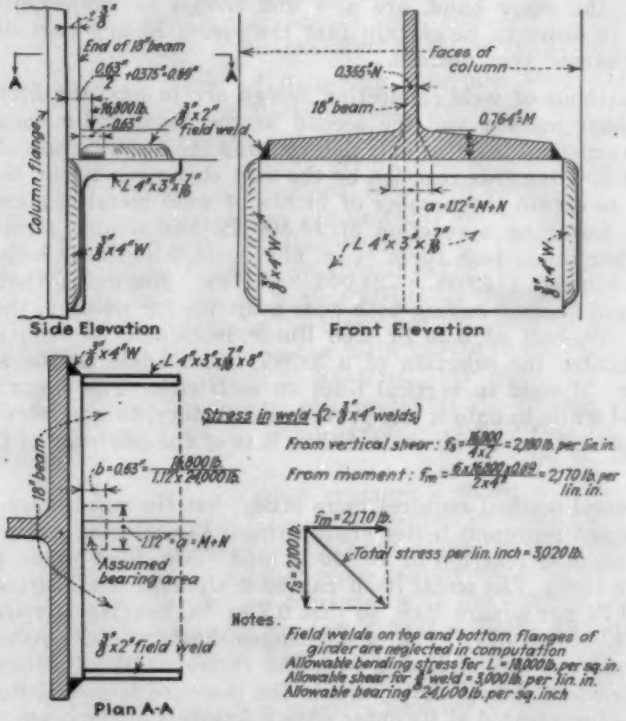

End of 18" beam
Column flange
16,800 lb.
3/8" x 2" field weld
L 4" x 3" x 7/16"
3/8" x 4" W
Side Elevation
Faces of column
0.355" = N
18" beam
0.764" = M
a = 1.12" = M + N
Front Elevation
Plan A-A
Assumed bearing area
Stress in weld (2-3/8" x 4" welds)
From vertical shear: fs = 16,800 / 4x2 = 2,100 lb. per lin. in.
From moment: fm = 6x16,800x0.69 / 2x4^2 = 2,170 lb. per lin. in.
fm = 2,170 lb.
fs = 2,100 lb.
Total stress per lin. inch = 3,020 lb.
Notes.
Field welds on top and bottom flanges of girder are neglected in computation
Allowable bending stress for L = 18,000 lb. per sq. in.
Allowable shear for 3/8" weld = 3,000 lb. per lin. in.
Allowable bearing 24,000 lb. per sq. inch

FIG. 1—DESIGNING A WELDED CONNECTION BY THE MOMENT-AND-SHEAR METHOD

From the first example, using the shear method, it appeared that a 3-in. vertical leg angle was required for shear only. Therefore, it is best to start the calculation by the shear-and-moment method by assuming an angle with a vertical leg of 4 in. The calculation for the stress in the weld at the extreme ends of welds is as follows:

$$\text{From shear, } f = \frac{16{,}800}{4\text{x}2 \text{ welds}} = 2{,}100 \text{ lb. per linear inch}$$

$$\text{From moment, } f = \frac{6\text{x}16{,}800\text{x}0.69}{2 \text{ welds x } (4)^2} = 2{,}175 \text{ lb. per linear inch}$$

Stress due to shear is vertical, while stress due to moment is horizontal.

Combining 2,100² plus 2,175² equals 3,020², or the maximum effect due to combined shear and moment is 3,020 lb. per linear inch of ⅜-in. fillet weld. It appears, therefore, that the connection is conservatively designed and that a 4x3x7/16-in. angle is required with 4 lin. in. of ⅜-in. fillet weld applied in vertical lines on both ends of the angle. When the beam has been erected it is well to apply 2 in. of ⅜-in. fillet weld to connect the edges of the beam to the angle. The second method described, which is illustrated in some detail in the accompanying drawing, is the better method of the two discussed, as the results are more conservative, and better knowledge of forces acting and stresses involved is provided. The second method offers the possibility of extension to cover various types of connections involving horizontal and vertical lines of welding. It also accounts for the higher test values shown by angle clips welded to columns by horizontal lines of welding instead of vertical lines. The second method deserves particular consideration in view of its wide range of application and the simplicity of the calculations.

Abstract Thesis "Studies of the Iron Arc"

C. A. STAY, E. S. PHELPS AND R. B. GEORGE*

Under the direction of Prof. F. Creedy, Electrical Engineering Department, Lehigh University

Editors Note: This thesis covers the results of experimental investigations of the Iron Arc made with low currents (up to 5 amps.). While the results are not directly applicable to the welding field, nevertheless they are of interest. The apparatus is so simple that it might be of interest to other investigators. Extracts from this thesis are, therefore, given below.

FOR obtaining data on the iron arc ordinary uncoated iron electrodes, 5/32 in. in diameter, were used. These were clamped in movable jaws so that the arc length could be adjusted at will. Current was at first supplied by means of a welding generator, with suitable resistance in the circuit to limit the current to from 1 to 5 amperes. An ammeter was used to measure the current and a voltmeter was connected across the two electrodes to measure the potential difference. An adjustable telescopic type magnifying lens was placed in front of the arc so that the magnified image of the arc appeared on the screen. The screen was placed to give a magnification of from 18 to 30 times actual size. A covering was used to exclude outside light from the screen and give a clear bright image. A sketch of this set-up is shown in Fig. 1. Later the equipment was transferred to the Packard Laboratory and the storage batteries were used to supply current. The batteries were found to give a more constant voltage and the results were better. The length of the arc was measured between the tips of the electrodes on the screen. Due to the uneven surfaces of the electrode tips while the arc is in operation it was thought that some error in measuring the arc lengths might occur due to parallax. To overcome parallax another magnifying lens was set up in a position at right angles to the first and the image thrown on a

*Senior Students, 1929-30.

mirror, from which it was reflected onto the screen. Measurements of the arc length on the two images thus obtained showed very slight differences, so it was decided that the effect of parallax was negligible. The magnification factor was found by placing an electric light behind the electrodes and measuring the actual rod diameter and the diameter of the projected image.

For study of the arc in nitrogen a mechanism for operating the electrodes was mounted on a wooden panel with the control rods coming through the back of the panel. A glass bell jar was then placed over the electrodes and held firmly in place by bolting it between two wooden panels. Inlet and outlet valves were provided in the panel at the open end of the bell jar so that nitrogen could be blown through the jar. Contacts connecting to the two electrodes were mounted on the outside

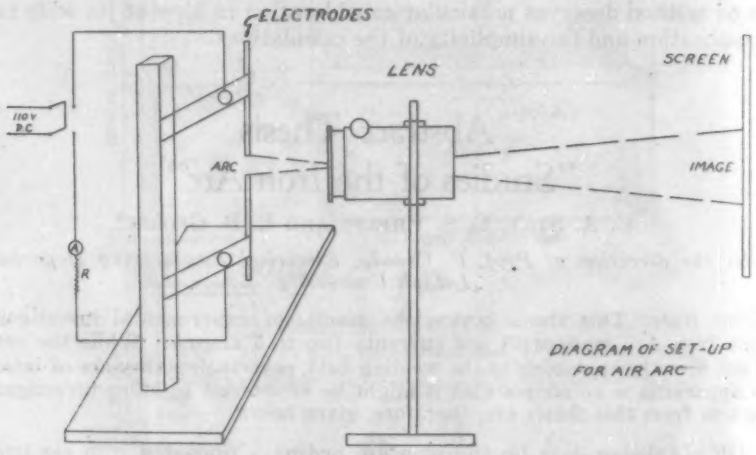


FIG. 1

of the panel. A diagram of this apparatus is shown in Fig. 2. The magnifying lens was placed outside the glass bell jar and focused so that the image of the arc was thrown on the screen.

For the constant current curves the current was held at a constant value and the arc length varied from the smallest amount measurable on the screen to the greatest length at which the arc could be maintained. The arc lengths were measured on the screen and the voltages across the arc noted for each arc length. It was found necessary to apply a correction to the measured voltage drops because the slag at the electrode tips caused a slight drop of voltage. This correction was found by placing the electrodes in contact after a run had been made and noting the voltage drop for different values of current. Since the corrections varied considerably from day to day an average value was used in applying the correction to the measured voltages.

Description of the Arc in Air

The magnified outlines of the arc were obtained by placing a sheet of paper on the screen and tracing the outlines of the image. The ends

of the electrodes are a bright orange in color due to the intense heat of the arc. The cathode spot is a small blue area near the tip of the cathode. This is surrounded by a light orange area, known as the reduction area, where the heat of the arc reduces iron oxide to pure iron. In observing these arcs it was necessary to use currents of less than 5 amperes since any larger current caused spattering of the arc, accompanied by rapid changes in length so that accurate observations were impossible. However, with the higher currents it was possible to observe the transfer of metal across the arc. The exact nature of this phenomenon is not known, but several theories have been advanced. One such theory is that the

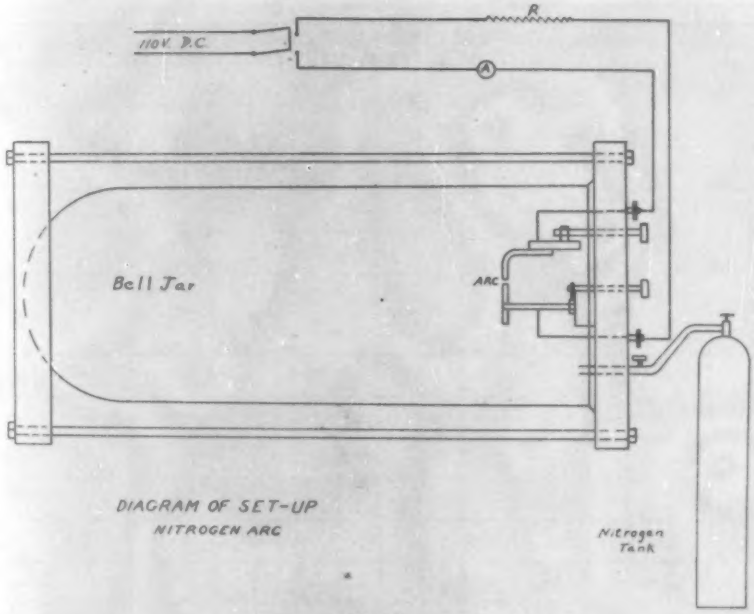


FIG. 2

gases liberated at the cathode form an iron bubble which is blown across the arc by the expansion of the gases from heat. The observations made in the experimental work on the iron arc tend to confirm this theory.

Observation of the arc when just sufficient current was flowing to cause a slow transfer of metal (about 10 amperes with 5/32 in. electrodes) showed very clearly that a gradual increase in size occurred at the tip of the cathode, similar to the formation of a bubble. This apparent bubble was unstable in size and shape, having the appearance of boiling. This agitation increased until it appeared to tremble violently, at which time it suddenly disappeared accompanied by a loud cracking sound and metal was transferred across the arc, to the accompaniment of spattering of the arc. These transitions of metal increase in rapidity as the current is increased until with the actual welding arc rapid fluctuations of current occur.

Description of the Arc in Nitrogen

The arc in nitrogen apparently does not rise to as high a temperature as the arc in air since the orange colored area at the tips of the electrodes does not spread as far. The cathode spot has the same appearance as in air, but there is no reduction area surrounding it, due to the absence of oxygen. At the tip of the cathode spot is a band of deep purple shading gradually to the bright blue of the core. The core in nitrogen is a much deeper shade of blue than in air and in the absence of oxygen the core is not surrounded by the yellowish striata observed in the air arc. The core shades off to a very light blue at the surface of the anode. The same blue white reflection of the cathode spot is observed on the anode.

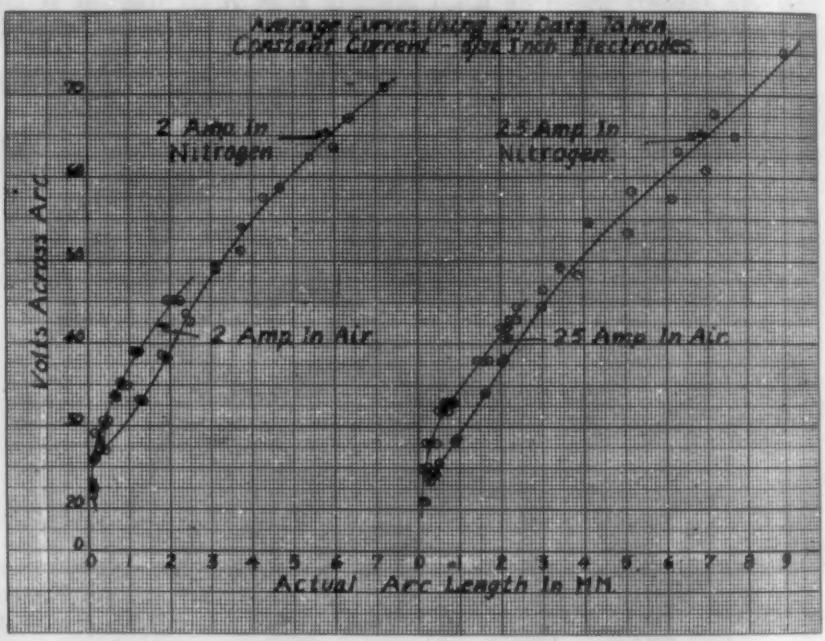

Average Curves Using All Data Taken
Constant Current - Pure Iron Electrodes
2 Amp in Nitrogen
2.5 Amp in Nitrogen
2 Amp in Air
2.5 Amp in Air
Volts Across Arc
Actual Arc Length in MM.

FIG. 3—CURVES A AND B

During the operation of the arc in nitrogen a heavy yellowish brown deposit formed on the walls of the bell jar. Analysis showed this to be iron particles which had been partly oxidized. These particles had apparently been thrown off from the anode since they were concentrated most thickly above the anode.

With the first tank of nitrogen used for this experimental work the arc was very stable and the arc length could be extended to about 9 millimeters without causing the arc to break down. With the second tank of nitrogen used an unstable arc was obtained in which the potential difference for a given length was less than before. With an arc in pure nitrogen the voltage drop was considerably less than in air, but the unstable condition caused this voltage drop to decrease still further.

Conclusions

The experimental work on the iron arc proves conclusively that the potential difference across the arc does not vary as a straight line function of the arc length nor as the first power of the current, either in air or in an inactive gas such as nitrogen.

The exact curves are shown in Fig. 3 and 4 A, B, C, D, E, which have been carefully checked. The tendency to instability in nitrogen is shown by the fact that the curves taken on different occasions do not agree so closely as in the case of air. Each of the curves in air is the mean of three sets of observations taken by different observers on different days.

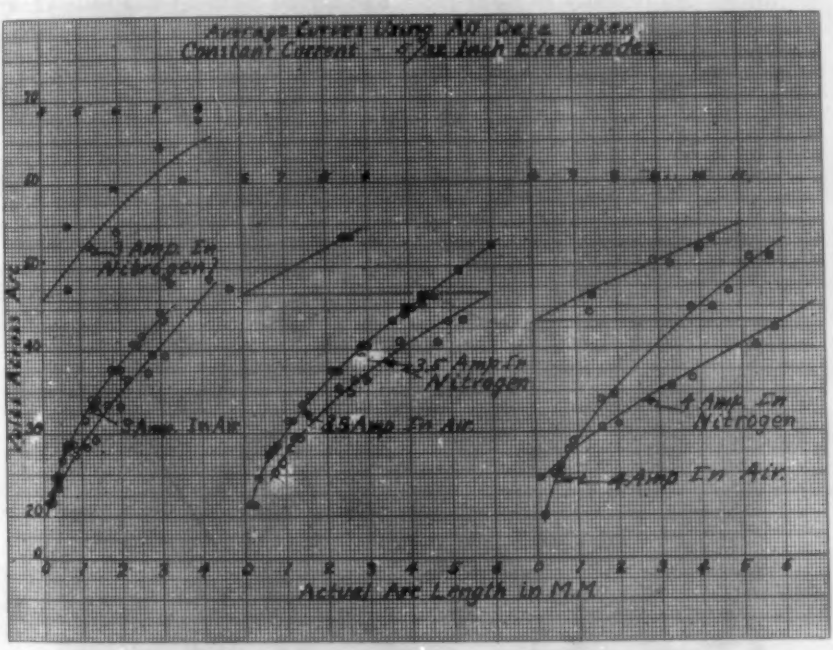

Average Curves Using All Data Taken
Constant Current - ¼ Per Inch Electrodes
3 Amp. In Nitrogen
3 Amp. In Air
3.5 Amp. In Nitrogen
3.5 Amp. In Air
4 Amp. In Nitrogen
4 Amp. In Air
Actual Arc Length in M.M.

FIG. 4—CURVES C, D AND E

Each of the curves in nitrogen is the mean of two such sets of observations. These curves may be very closely represented by the following equations:

The Iron Arc in Air

$$\text{Volts} = 48i^{-.35}\, l^{.1851^{\cdot 35}}$$

The Iron Arc in Nitrogen

$$\text{Volts} = 35i^{-.21}\, l^{.881^{\cdot 21}}$$

i = current in amperes. l = arc length in millimeters.

A considerable number of values of the four numerical coefficients involved can be found which all fit the curves nearly equally well. This is due to the fact that a given variation in one coefficient may be compensated for by a corresponding variation in another.

The Progress of Bronze Welding

DEW. ENDICOTT†

THE constantly increasing use of bronze-welding throughout all industries for making joints in nearly all ferrous and non-ferrous metals makes it desirable to discuss in some detail the uses of this application. As recently as eight years ago bronze-welding was little known outside of a very few large industries. Today practically all commercial metals are being bronze-welded, the size of the work imposing no limitation on the use of this process. It is even interesting to compare the work being done by this method only three years ago with the tremendous variety of work being done today.

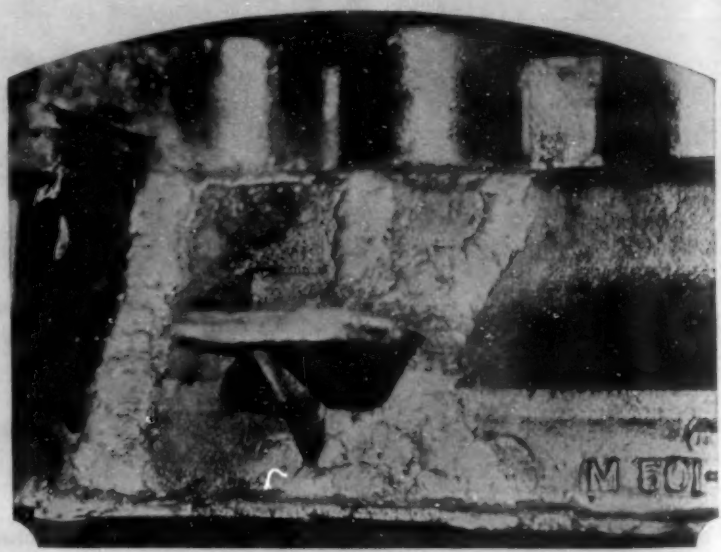


FIG. 1—AUTOMOBILE CYLINDER BLOCK BROKEN IN THIRTEEN PLACES RESTORED BY BRONZE-WELDING

Bronze-welding should not be confused with brazing, although the process is somewhat similar. Brazing, which metal-workers have employed for a long time, consists in heating the metal parts to be joined hot enough to melt a brass alloy called "spelter," usually supplied in powder or chip form, so that it flows into the fluxed joint. The use of the oxy-acetylene flame to accomplish brazing led to the development of the modern art of bronze-welding. In bronze-welding, the bronze is supplied in the form of welding rod and the oxy-acetylene flame serves to heat the base metal to the proper temperature as well as to melt the

†Technical Publicity Dept., Linde Air Products Co.

welding rod. The operator is thus enabled to control the work accurately at all times. Although not a fusion weld, in many cases a bronze-weld made with a high grade bronze-welding rod and good flux produces a joint comparable in strength and reliability to a fusion weld.

Bronze-welding is extensively used today both in repair and production operations. It is used in three general ways: for producing joints in metals with a higher melting point than bronze, which includes nearly all commercial metals; for building up worn surfaces and for making fusion welds in bronze itself.

FIG. 2—LIMESTONE CRUSHER IN CEMENT PLANT. A LARGE BRONZE-WELDED CAST IRON REPAIR

Cast Iron

The bronze-welded joint made in cast iron is the most familiar application of this process. This application is especially advantageous as the joint can be quickly made with the application of very little heat. In bronze-welding cast iron general preheating in a furnace is seldom necessary. Some preheating may be required for large castings, but it is usually possible to arrange some means of supplying the heat locally, as only the parts in the immediate vicinity of the break need to be heated. This means that large castings need not be dismantled but may be welded in their operating positions. Frequently the entire cost of a bronze-welded repair is far less than just the labor cost of dismantling the machine to permit repair by any other method.

For small castings, the welding blowpipe itself is used as a convenient means of preheating the part before welding.

Cast iron should be prepared for bronze-welding by chipping or grinding the surfaces to be joined to form a vee; cleaning them thoroughly after grinding so that the bronze will have a clean surface to which to adhere. After the parts are aligned in the correct positions, the blowpipe flame should be played on the metal surrounding the break so that it is heated gradually to the point where it begins to glow. If the cast iron is being preheated for bronze-welding, it should be heated only to a black heat, which is the point just below that of the dull red required for bronze-welding. A casting hot enough to char a crumpled newspaper or sawdust is ready to weld with bronze. However, the surface being welded must be brought by the blowpipe to the proper temperature, as, if it is too hot the bronze will boil and run off like mercury and if it is not hot enough, the bronze will not adhere.

FIG. 3—DIESEL ENGINE FRAME REPAIRED IN TUG

The use of a high strength bronze welding rod and suitable flux is essential to successful bronze-welding. While the metal of the casting is being heated by the blowpipe flame the end of the bronze rod should be heated and the hot end dipped in the can of flux so that sufficient flux will adhere to the rod. The end of the fluxed rod should be melted onto the heated spot. With the metal at the proper temperature, the molten bronze will flow in a thin layer and spread out over the metal ahead of the main deposit to produce the tinning coating. Proper tinning is a vitally important factor in the technique for bronze-welding cast iron. After the tinning layer has been obtained, more rod can be added rapidly and the vee quickly built up. A section of the metal about 2 in. along the vee is heated and the weld made in the above manner, first tinning, then building up. The next increment is then made. Fluxing should be continued

throughout the operation so that any impurities or bubbles of gas in the weld metal will be removed. As in all ripple welding a swinging motion with the blowpipe is used, care being taken that the inner cone of the blowpipe flame is kept between ⅛ and ¼ in. away from the surface of the puddle.

It is advantageous for a bronze-weld to cool slowly and in the case of a large casting a sheet of asbestos paper is often used to cover the finished job and protect it from drafts during the cooling. No stress should be placed on a bronze-welded joint until it has completely cooled. The finished weld may be cleaned with a wire brush to remove any excess flux or impurities which may have risen to the surface.

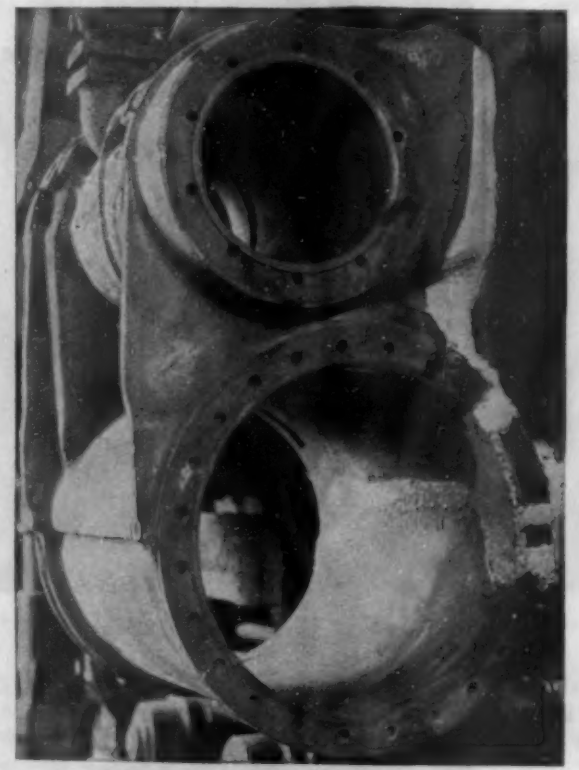


FIG. 4—STEAM CYLINDERS OF INDUSTRIAL LOCOMOTIVES ARE BRONZE-WELDED

The bronze-welding of cast iron has been a standard practice in the field of automotive repair. Cast iron cylinder blocks are regularly repaired by bronze-welding by the oxy-acetylene process. Nearly all automobile repair shops today are familiar with the practice of bronze-welding cast iron. An automobile cylinder block may be bronze-welded in place, although it is recommended that if convenient the block be removed and placed in a position in which it may be most easily welded. If the

block is left in the car, all other parts of the machinery must be protected from the heat of the oxy-acetylene flame, and all gasoline thoroughly drained. A variety of types of cracks occur in cylinder blocks. Often the crack is in the water jacket in the head, in the cylinder wall, in the corners of the block, or in the cast iron arm. The water circulation system should be drained below the level of the crack and the cooling system vented so that no pressure will be built up from the heat. Some shops make a practice of running the motor for a few minutes with the cooling system drained so that the casting will be preheated to a certain

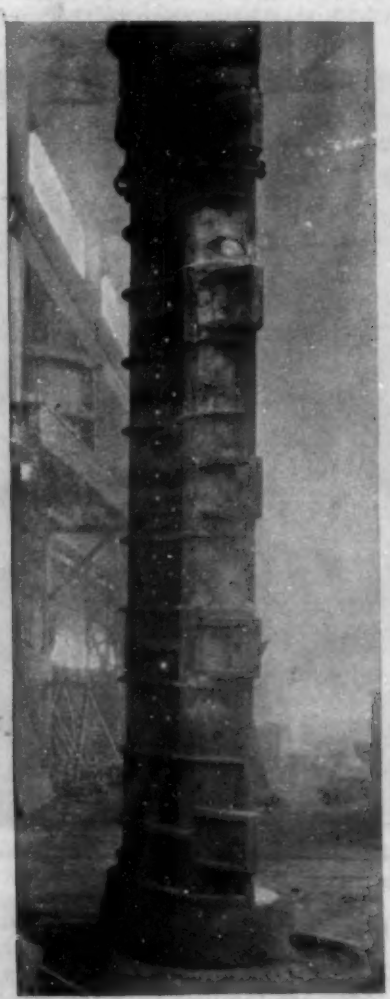


FIG. 5—CRACK SIX FEET LONG IN LARGE CAST IRON PIPE FOUNDRY FLASK BRONZE-WELDED BY OXY-ACETYLENE PROCESS

extent. As in all cases of the bronze-welding of cast iron, the crack should be chipped out throughout its entire length and slightly beyond the extremes of both ends where the crack may not be evident on the surface of the metal. The same general rules for bronze-welding apply in repairing cylinder blocks as in other cast iron repairs. Where a piece of the cast iron is completely broken out of the block, it should be tack-welded in place with the bronze-welding rod before the final bronze-welding is accomplished.

Within the last two years the bronze-welding of cast iron has been extended to pieces of machinery of tremendous size, many of which would formerly have been considered unweldable. Large presses which often become fractured through overloading are regularly repaired in bronze-welding. Rock crushers or limestone crushers used in rock products and

FIG. 6—SHEAR VEE JOINT IN CAST IRON PIPE WITH BRONZE-WELD PARTIALLY COMPLETED

cement plants have been among the more recent bronze-welding repairs made by the oxy-acetylene process. When bronze-welding repairs of this size are being made it is frequently necessary to have gas torch preheating flames constantly playing on the cast iron which is to be bronze-welded in order to maintain the proper heat. In jobs of this size repairs would be practically impossible were it not for bronze-welding, as the difficulties of preheating castings of this size in a furnace would be tremendous. In a number of cases within the last few months, enormous cast iron power shears used for cutting steel billets have had the side frames cracked under the strain and have been restored to service by bronze-welding. Repairs to Diesel engine frames by means of bronze-welding are also becoming more common. In one particular case a cracked Diesel engine frame was repaired in a tug without removing any of the

parts to the shop, the welding being accomplished below the water line of the tug. If bronze-welding had not been employed a complete dismantling of the engine would have been required. Many castings of unusual size, such as are used on underground pumps in mines have been saved from the scrap pile by this application.

Industrial type locomotives are used extensively by large manufacturing plants, mines and lumber companies and bronze-welding has proved to be invaluable in keeping this equipment in service.

Cast iron foundry flasks are frequently broken and it is only in recent years that the foundries have realized the tremendous savings which may be effected by reclaiming them. Formerly these flasks were scrapped and the metal remelted but today the speed and economy of bronze-welding have led to its use by a great number of foundries. These flasks receive particularly hard service and, after each pouring, the mold is frequently dropped several feet to clean it so that breakage is quite frequent. Intricately shaped flasks for thin castings are particularly valuable and worth reclaiming. Foundry welding departments are called upon to bronze-weld long seams and also patches. Sometimes these cracks extend for 6 ft. or more. In small flasks breakage occurs frequently by enlargement of the hole. These holes are now filled with bronze and redrilled. A bronze bushing of this type will never work loose. The savings which foundries have effected by this means of reclamation are enormous.

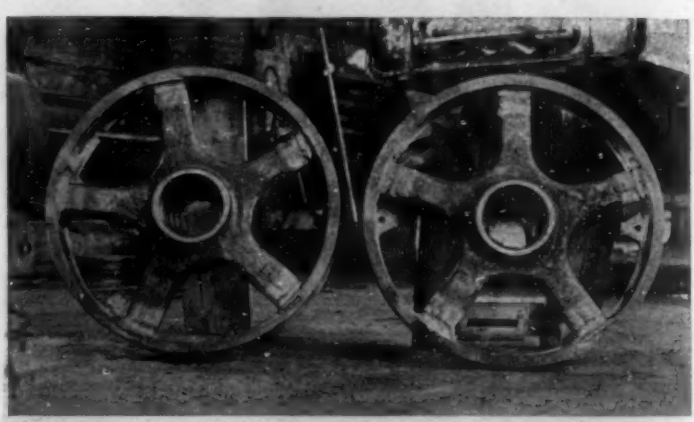


FIG. 7—NEW STEEL RIMS BRONZE-WELDED TO CAST STEEL TRUCK WHEEL SPOKES

During this year an increasing number of water pipe lines have been constructed of cast iron pipe with bronze-welded joints. The bronze-welded joint of the new improved shear-vee type, made under procedure control methods, is free from leakage and future maintenance expense. Municipal water departments in all parts of the country are realizing the value of this application. Not only is bronze-welding employed to make joints in cast iron pipe on pipe lines but also the standard 12-ft. lengths

of cast iron pipe are double-lengthed into 24-ft. sections at the foundry by bronze-welding before they are shipped to the job. This practice is recommended because it reduces the number of field joints to be made and facilitates transportation. In certain cases, cast iron pipe has been triple-lengthed at the foundry and shipped in 36-ft. sections. When aligning the pipe for welding, the joint should be tack-welded in three places, and, when the actual welding is being done, the puddle should be started and maintained near the top quarter of the circumference, so that the pipe has to be rotated during welding. It may well be emphasized again that the bronze-welding of cast iron pipe should be done only in accordance with procedure control.

FIG. 8—CLOSEUP OF BRONZE-WELD IN GALVANIZED IRON PIPE

Malleable Iron

Bronze-welding is the only successful method of making strong joints in malleable iron castings. If malleable iron is melted, its malleable properties are completely destroyed and it is almost impossible for the user to restore them as they are the result of a special heat treatment, the facilities for which are available only at iron foundries and at special shops. The changes brought about in this metal by fusion practically require that a method of joining be employed which does not involve bringing the base metal to the melting point. The comparatively low temperature required for bronze-welding has no harmful effect on malleable castings, and this method is therefore employed for making welds in this metal. Malleable castings are usually small in size and seldom require preheating. The technique for bronze-welding this metal is the same as that for bronze-welding cast iron. A great deal of money is saved annually by textile mills and other industries which use a large number of malleable castings, through reclamation by bronze-welding.

Steel and Wrought Iron

The technique for bronze-welding steel or wrought iron is similar to that for making joints in cast iron. The metal should be heated by the blowpipe to a dull red heat before the bronze is flowed onto the joint. Bronze-welding is frequently found advantageous in production operations for making joints in steel where the strength of a fusion weld using steel welding rod is not required and where the color of the joint is immaterial. In the production of metal furniture there is an outstanding example of the use of bronze-welding of steel. Bronze-welding may be accomplished with much greater speed than steel welding with a consequent saving in oxygen and acetylene. Another factor in favor of bronze-welding on light sheet steel is that there is less opportunity for warpage

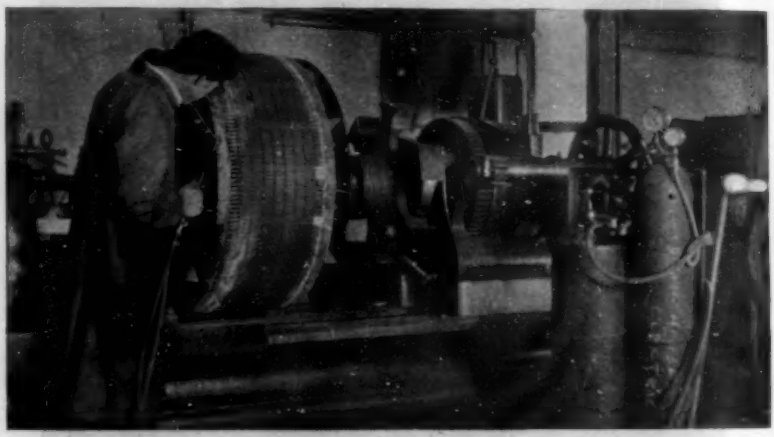


FIG. 9—COPPER BARS BRONZE-WELDED TO BRONZE END RINGS OF ROTOR FOR 250-HP. ELECTRIC MOTOR

because less heat is required to accomplish the weld. In fact, bronze-welding is nearly always employed in the production of metal furniture except where a vitreous enamel is required and in that case a steel weld is used in order to withstand the enameling temperature.

Bronze-welding is used on wrought iron in ornamental iron work. With this process the metal artist need not bring the part to be joined to the melting point. The bronze will flow between tight fitting metal parts and the temperature required is not high enough to mar any sharp edges. In some cases of this type a fusion weld in the wrought iron would be impossible without warping or completely melting the objects being welded.

Galvanized Iron

The welding of galvanized iron presents somewhat of a problem owing to the fact that the coating of zinc which forms the galvanizing has a relatively low melting point and also has the property of volatilizing when heated. When the galvanized material is heated, the zinc coating does not melt and run on the surface but vaporizes and burns, forming zinc

oxide fumes. When a fusion weld using steel welding rod is attempted on galvanized iron, the galvanized coating is usually burned for about an inch on each side of the weld so that it may be necessary to apply a protective coating over the section near the weld after the joint is made. The low melting point of high strength bronze-welding rod makes it possible to make the joint at a temperature low enough so that the galvanizing on each side of the weld is not entirely removed. Galvanized iron pipe which is frequently used for water and other industrial services may best be bronze-welded and the completed weld will require no coating of paint or other protective material afterward. Many manufacturers of large storage tanks made of galvanized iron have adopted bronze-welding as a regular production process, as they have found it the most economical way of obtaining a satisfactory joint.

FIG. 10—BRASS WATER PIPE FOR BUILDINGS REQUIRES PERMANENCY

Copper

For a number of years very little welding was done on copper. In order to obtain successful fusion welds in this metal it is necessary to have specially deoxidized copper sheets and to use a copper rod with a sufficient amount of silicon to insure complete freedom from oxide. Although it is now possible to obtain copper stock specially deoxidized with silicon for welding, a greater part of copper joining consists in work on ordinary commercial copper. For such work it has been found that bronze-welding will produce a strong, tight joint free from corrosion. The technique

for bronze-welding pipe, castings, or sheets of copper is the same as for bronze-welding the ferrous metals. In the case of copper, however, care must be taken to use as little heat as necessary on the base metal, as copper has a high rate of thermal conductivity, so that the heat is soon dissipated through the surrounding metal. Bronze-welding must therefore be accomplished quickly and the copper should not be heated any further back from the weld than is absolutely necessary. Copper cast-

FIG. 11—FUSION WELDS IN BRONZE ARE BEST MADE WITH BRONZE WELDING ROD

ings and material over ⅛ in. thick should have the edges beveled for welding. Proper tinning is as important to the bronze-welding of copper as it is in the case of cast iron or steel. The parts to be joined should be heated to a temperature which will show a red heat in daylight which will be the point at which the bronze will unite to the base metal.

This application is particularly useful in the fabrication of chemical containers of copper and also in the manufacture of water pipes, and hot water heating coils. Copper sheet, pipe and castings are being used

to a greater extent today than ever before in a number of industries and the old fashioned mechanical joint, folded and grooved, riveted or screw-coupled has been inadequate where strength and tightness are desired. The fact that bronze-welding is able to produce joints capable of satisfying the demand of these industries for tight joints in commercial copper has led to a widespread use of this application.

Brass and Bronze

Brass water piping in buildings has been the chief application of the bronze-welding of this metal. When brass is melted the metal boils and

FIG. 12—ROTARY VALVES ON PUMPING EQUIPMENT BUILT UP WITH MANGANESE BRONZE

bubbles at the surface and gives off zinc oxide fumes, indicating a loss of zinc with a consequent change in the character of the metal near the weld. It is therefore advantageous to use a bronze-welding rod with a low melting point to produce a bronze-weld in this metal. Many large buildings recently have had complete water piping systems installed of brass with bronze-welded joints.

Bronze-welding is frequently employed to make fusion welds in bronze castings. In these cases the castings are usually preheated completely and the heat retained by an asbestos covering during the welding. By using a high strength bronze welding rod with a low melting point, it is necessary only to bring the base metal up to the point of fusion which will avoid any possibility of any fuming or boiling of the base metal during the welding.

For Building Up Worn Surfaces

Bronze-welding is being used effectively as a method of building up wearing surfaces. When the part again becomes worn more bronze may be added to the surface to restore it to normal dimensions. Where the wearing action is particularly severe, manganese bronze welding rod is advantageously employed. Examples of this have been found in the building up of Diesel engine pistons. When these pistons become badly worn, it would be necessary to discard both the piston and the cylinder if it were not possible to build up the piston to its right size. Bronze-welding is used to build up the pistons which are subsequently machined down to the correct size. Oil companies build up rotary valves and machine them to fit the rebored cylinders of pumping engines. A manganese bronze welding rod is usually employed in building up rotary valves of this type. This application is proving extremely valuable in restoring many parts of machinery.

Conclusion

From this brief outline it may be seen that bronze-welding is an application almost without limit. Practically all the commercial metals may be bronze-welded, even nickel and Monel have been successfully bronze-welded. The progress of this application during the last year has been so rapid that there are few users of the oxy-acetylene process who do not realize the value of this operation.

Research and instruction have been two of the main factors in promoting the use of bronze-welding. The development of new bronze welding rods of higher strength and a melting point as low as 1638 deg. F. has been an outstanding factor in advancing this application during this year. High strength bronze-welding rod also possesses the advantage of freedom from extensive fuming during the welding.

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